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# USING INSTRUMENTED VEHICLES TO DETECT DAMAGE IN BRIDGES

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## ABSTRACT

Bridge structures are subject to continuous degradation due to the environment, ageing and excess loading. Monitoring of bridges is a key part of any maintenance strategy as it can give early warning if a bridge is becoming unsafe. This paper will theoretically assess the ability of a vehicle fitted with accelerometers on its axles to detect changes in damping of bridges, which may be the result of damage. Two vehicle models are used in this investigation. The first is a two degree-of-freedom quarter-car and the second is a four degree-of-freedom half-car. The bridge is modelled as a simply supported beam and the interaction between the vehicle and the bridge is a coupled dynamic interaction algorithm. Both smooth and rough road profiles are used in the simulation and results indicate that changes in bridge damping can be detected by the vehicle models for a range of vehicle velocities and bridge spans.

**KEY WORDS** Bridge, damage, vehicle, accelerometer, damping, frequency

## INTRODUCTION

In the past, the task of detecting damage in bridges was done by visual inspection, which is a labour intensive and often inaccurate method of determining the true condition of a bridge. With the increase in computational power and signal processing capacity, there is a move towards sensor based analysis of the condition of bridges. Changes in the physical properties of a bridge (stiffness, mass and energy dissipation mechanisms) cause changes in the spectral properties of that bridge (frequencies, damping and mode shapes). For example, a change in the stiffness of a bridge (indicating that the bridge is damaged) can be detected by a change in the natural frequency of the bridge. The maintenance of bridges (involving sensors) initially involved the direct instrumentation of the bridge – commonly referred to as Structural Health Monitoring (SHM) (Brownjohn, 2007, Chang et al., 2003, Farrar & Worden, 2007). However, monitoring the bridge via direct instrumentation requires installation and maintenance of sensors and data acquisition electronics on the bridge which can be expensive and time consuming. More recently, there has been a move towards instrumentation of a vehicle, rather than the bridge in order to detect damage in the bridge.

Many authors use the observation of natural frequency change as a damage detection mechanism as natural frequencies are sensitive indicators of structural integrity. They can also be measured with relative ease, and are inexpensive to measure (Carden & Fanning, 2004, Salawu, 1997). The feasibility of detecting frequencies from the dynamic response of an instrumented vehicle passing over a bridge has been verified theoretically by (McGetrick et al., 2009, Yang et al., 2004). The method proposed by (Yang et al., 2004) was later tested in field trials (Lin & Yang, 2005, Yang & Chang, 2009). Experimental investigations have also been conducted to check the feasibility of the approach as part of a drive-by inspection system for bridge monitoring (Kim & Kawatani, 2009, Oshima et al., 2008, Toshinami et al., 2010). It should be noted however that using frequency shifts to detect damage has practical

limitations, especially in the case of large structures (Curadelli et al., 2008). A numerical and experimental study by (González et al., 2008) analysed a 3D FEM vehicle-bridge interaction model and they concluded that accurate determination of the bridge frequency is only feasible for low velocities and high dynamic excitation of the bridge.

The analysis of damping has been considered to a lesser extent than natural frequencies, in the field of damage detection, due to difficulty in quantifying its magnitude (Williams & Salawu, 1997). However recent evidence suggests that damping is quite sensitive to damage in structural elements and in some cases, more sensitive than natural frequencies. (Curadelli et al., 2008) show that when cracks result in little or no frequency variation, changes in damping may be used to detect the nonlinear dissipative effects that cracks produce. (Modena et al., 1999) show that visibly undetectable cracks cause very little change in resonant frequencies and require higher mode shapes to be detected, while these same cracks cause larger changes in damping. In some cases, damping changes of around 50% were observed. Many other authors have noted that damping is highly indicative of the amount of damage that a structure has undergone during its lifetime (Gutenbrunner et al., 2007, Jeary, 1996, Kawiecki, 2001).

Given the move in the direction of sensor based analysis of the condition of bridges, and the importance of theoretical computer modelling, the issue of accurate vehicle modelling, accurate bridge modelling and modelling of their interaction has come to the fore. Some authors have modelled the vehicle as a single vertical force or as a series of constant forces (Brady & OBrien, 2006, Brady et al., 2006, Savin, 2001). Other authors have modelled the vehicle as a lumped sprung mass model (Green et al., 1995, Yang & Chang, 2009, Yang & Lin, 2005, Yang et al., 2004) or a train as a series of sprung masses lumped at the bogie positions (Yang & Yau, 1997). A more comprehensive vehicle model is the two degree-of-freedom (vehicle bounce and axle hop) quarter-car used by many authors (Green & Cebon, 1994, Li et al., 2006, McGetrick et al., 2009, Seetapan & Chucheeepsakul, 2006, Yang et al., 1999). However, quarter-cars lack the capability to simulate the pitching effect of the car body on the vehicle responses. A further development of the two degree-of-freedom quarter-car, is the four degree-of-freedom half-car (Cebon, 1999, Green & Cebon, 1994, Kim et al., 2005, OBrien et al., 2006). Some authors have extended this further and have modelled an articulated tractor-trailer (Cantero et al., 2010, González et al., 2008, Harris et al., 2007, Nassif & Liu, 2004, OBrien et al., 2010). Other authors have modelled real world vehicles theoretically – such as the Ford Cargo truck and the Isuzu dump truck (Kim et al., 2005) and the AASHTO HS20-44 truck (Deng & Cai, 2010).

Many authors have modelled the bridge as a simply supported beam (Brady & OBrien, 2006, Brady et al., 2006, Green & Cebon, 1994, Green & Cebon, 1997, Harris et al., 2007, Law & Zhu, 2005, Li et al., 2006, Liu et al., 2002, McGetrick et al., 2009, OBrien et al., 2006, Yang & Chang, 2009, Yang & Lin, 2005, Yang et al., 2004, Zhang et al., 2001). Although the simply supported bridge model is not representative of all types of bridge, it embodies many of the important dynamic characteristics (Green & Cebon, 1997). Some authors have modelled the bridge as a multi-span continuous orthotropic rectangular plate (Zhu & Law, 2002), as a two dimensional eight-noded quadratic plate/shell element (Fafard et al., 1997), as a two-span cellular bridge with edge cantilevers (González et al., 2008) or as I beam sections with a cast in place deck (Liu et al., 2002). Other authors have opted to model real world bridges, such as the Mura River Bridge in Slovenia (González et al., 2008) and the Da-Wu-Lun bridge near the northern coast of Taiwan (Lin & Yang, 2005).

To model the interaction between the vehicle and the bridge, three types of algorithm may be used. The first method is the direct time integration method (Deng & Cai, 2010, Green et al.,

1995, Henchi et al., 1998, Kim et al., 2005, Law & Zhu, 2005, Seetapan & Chucheeepsakul, 2006, Yang & Chang, 2009, Zhu & Law, 2002). It involves the construction of a set of coupled equations with a large number of degrees-of-freedom for both the beam and the load system, the solution of which may found without iteration. The second method is the iterative method (Green & Cebon, 1994, Harris et al., 2007). In this method, two uncoupled sets of equations, one for the beam and one for the loads are formulated separately with compatibility conditions and equilibrium conditions for the interaction forces between the beam and the load system. The third method (Yang & Fonder, 1996, Yang et al., 1999, Yang & Lin, 1995, Yang & Yau, 1997) involves the definition of an interaction element. The whole bridge-vehicle system is divided into two subsystems at the interface of the bridge and the vehicles, where the interaction element is defined. The element consists of a beam element and the suspension units of the load resting on the element and all the degrees-of-freedom associated with the load system within each substructure are eliminated by dynamic condensation.

This paper assesses the ability of a vehicle fitted with accelerometers on its axles to detect changes in damping of bridges. Two vehicle models are used in this investigation. The first is a two degree-of-freedom quarter-car and the second is a four degree-of-freedom half-car, as described in the section below. The bridge is modelled as a simply supported beam and the interaction between the vehicle and the bridge is a coupled algorithm.

## VEHICLE – BRIDGE INTERACTION MODEL

Two different vehicle models are used in this study to represent the behaviour of the vehicle. The first is a theoretical quarter-car model (Fig. 1) with two degrees-of-freedom, which allows for axle hop and sprung mass bounce. The body of the vehicle is represented by the sprung mass,  $m_s$ , and the axle component is represented by the unsprung mass  $m_u$ . The body mass is connected to the tyre by a spring of stiffness  $K_s$  and a viscous damper  $C_s$ . The axle mass connects to the road surface via a spring of stiffness  $K_t$ . Tyre damping is assumed negligible and is thus omitted. The model also accounts for the sprung mass moment of inertia,  $I_s$ .

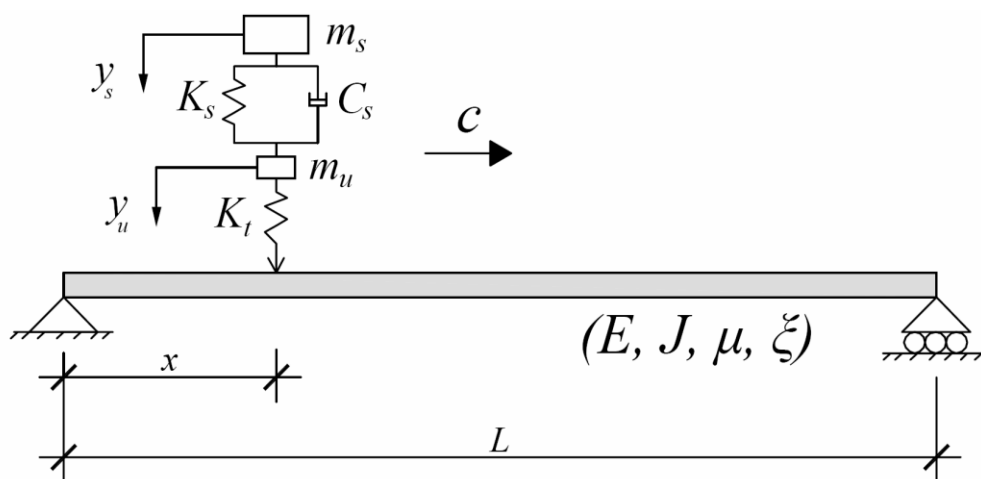


Fig. 1 Theoretical quarter car model

The properties of this vehicle model are listed in Table 1 and are based on values obtained from work by (Cebon, 1999, Harris et al., 2007).

Table 1 Properties of the quarter car vehicle model

| Property                         | Unit         | Symbol              | Quarter Car Model  |
|----------------------------------|--------------|---------------------|--------------------|
| Body mass                        | kg           | $m_s$               | 17 300             |
| Axle mass                        | kg           | $m_u$               | 700                |
| Suspension Stiffness             | N/m          | $K_s$               | $4 \times 10^5$    |
| Suspension Damping               | N s/m        | $C_s$               | $10 \times 10^3$   |
| Tyre Stiffness                   | N/m          | $K_t$               | $1.75 \times 10^6$ |
| Moment of Inertia                | $\text{m}^4$ | $I_s$               | 99 803             |
| Body mass frequency of vibration | Hz           | $f_{\text{bounce}}$ | 0.69               |
| Axle mass frequency of vibration | Hz           | $f_{\text{axle}}$   | 8.8                |

The second vehicle model is a theoretical half-car model (Fig. 2) with four degrees-of-freedom, which allows for sprung mass bounce, sprung mass pitch rotation and axle hop of each axle. The body of the vehicle is represented by the sprung mass,  $m_s$ , and the axle components are represented by the unsprung masses  $m_{u1}$  and  $m_{u2}$ . The body mass is connected to the tyres by springs of stiffness  $K_{s1}$  and  $K_{s2}$  and viscous dampers  $C_{s1}$  and  $C_{s2}$ . The axle masses connect to the road surface via springs of stiffness  $K_{t1}$  and  $K_{t2}$ . Tyre damping is also assumed negligible here and is thus omitted. The model also accounts for the sprung mass moment of inertia,  $I_s$ , and the distance of each axle to the vehicle's centre of gravity, i.e.,  $D_1$  and  $D_2$  in Table 2. The centre of gravity of the vehicle is taken to be equidistant from each axle ( $D_1 = D_2$ ), i.e., body weight equally distributed between axles.

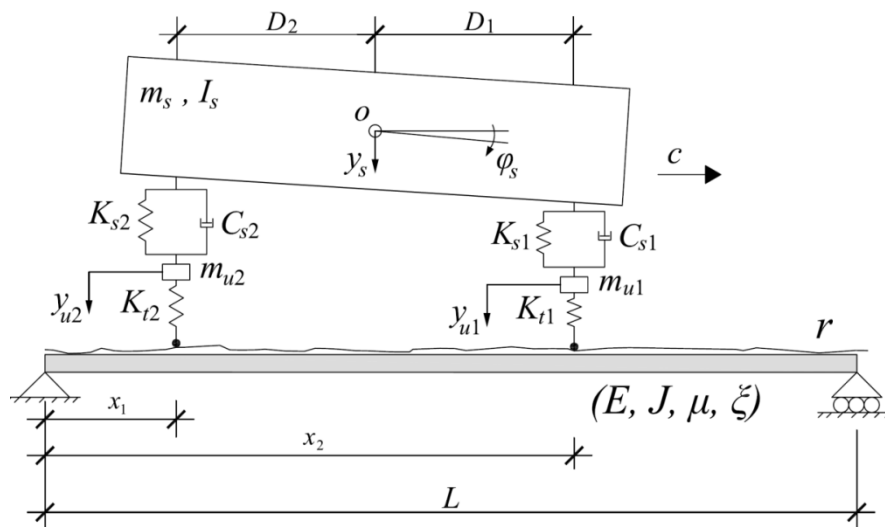


Fig. 2 Theoretical half car model

The half car vehicle properties are listed in Table 2 and are based on values obtained from work by (Cebon, 1999, Harris et al., 2007). It should be noted that the properties of the front and rear axles are the same.

Table 2 Properties of the quarter car vehicle model

| Property                              | Unit  | Symbol                 | Half-car Model Equal axles |
|---------------------------------------|-------|------------------------|----------------------------|
| Body mass                             | kg    | $m_s$                  | 16600                      |
| Axle mass                             | kg    | $m_{u1}, m_{u2}$       | 700                        |
| Suspension Stiffness                  | N/m   | $K_{s1}, K_{s2}$       | $4 \times 10^5$            |
| Suspension Damping                    | N s/m | $C_{s1}, C_{s2}$       | $10 \times 10^3$           |
| Tyre Stiffness                        | N/m   | $K_{t1}, K_{t2}$       | $1.75 \times 10^6$         |
| Moment of Inertia                     | $m^4$ | $I_s$                  | 95765                      |
| Distance of axle to centre of gravity | m     | $D_1, D_2$             | 2.375                      |
| Body mass frequency of vibration      | Hz    | $f_{bounce}$           | 1.0                        |
|                                       |       | $f_{pitch}$            | 1.0                        |
| Axle mass frequency of vibration      | Hz    | $f_{axle1}, f_{axle2}$ | 8.8                        |

Each vehicle model travels over a simply supported Euler-Bernoulli beam. The bridge span is varied from 10 m to 15 m, 25 m and 35 m. The 10 m and 15m bridge models have a T-beam cross section, the 25 m bridge model has a Y-beam cross section and the 35 m bridge model has a super Y-beam cross section. Each bridge model has a constant modulus of elasticity  $E = 3.5 \times 10^{10}$  N/m<sup>2</sup>. The mass per unit length,  $\mu$ , and the second moment of area  $J$  of each bridge model are given in Table 3. Structural damping varies from 0% to 5% and the speed of the vehicle is varied in simulations from 20 m/s to 22.5 m/s and 25 m/s. Prior knowledge of the first natural frequencies of the bridge are assumed here and are given in Table 3.

Table 3 Properties of the bridge models

| Bridge Span                    | 10 m                   | 15 m                  | 25 m                  | 35 m                  |
|--------------------------------|------------------------|-----------------------|-----------------------|-----------------------|
| First Natural Frequency        | 8.6 Hz                 | 5.65 Hz               | 4 Hz                  | 3 Hz                  |
| Mass per unit length ( $\mu$ ) | 18 750 kg/m            | 28 125 kg/m           | 18 358 kg/m           | 21 752 kg/m           |
| Second moment of area ( $J$ )  | 0.16095 m <sup>4</sup> | 0.5273 m <sup>4</sup> | 1.3901 m <sup>4</sup> | 3.4162 m <sup>4</sup> |

The crossing of the vehicle models over each bridge is described by a system of coupled differential equations and is based on the approach proposed by (Frýba, 1999). The system of equations is solved using the Wilson-Theta integration scheme (Tedesco et al., 1999). The value of  $\theta$  used is 1.420815 (Weaver & Johnston, 1987).

The vehicle models are simulated crossing both smooth and rough road profiles to assess their potential to detect damage in the bridge. Initially the quarter-car model is simulated running over a smooth profile and then over a rough profile. Secondly, the half-car model is simulated running over a rough profile. The results may be seen in the following section.

## RESULTS OF THE QUARTER CAR SIMULATIONS

The simulations in this section are performed using the vehicle-bridge interaction model outlined in the above section with a smooth road surface profile. The scanning frequency used in all simulations is 1000 Hz. The properties varied in these simulations are the bridge span, bridge structural damping and vehicle velocity. Having already obtained the bridge frequencies (Table 3), one can locate and differentiate between them in the acceleration spectra and compare recorded frequencies to the tabulated values.

The power spectrum of the processed accelerations of the vehicle model travelling at 20 m/s over a 10 m bridge can be seen in Fig. 3. The spectra of accelerations obtained at all the simulated damping levels are also illustrated. A clear peak is visible at 9.76 Hz which is near to but not equal to the first natural frequency of the bridge. The inaccuracy is due to the resolution of the spectra ( $\pm 1.95$ ), which can be improved by driving the vehicle at a slower speed. Significantly, the magnitude of the peak decreases for higher levels of damping. Hence a change in damping as a result of bridge damage, can be easily detected.

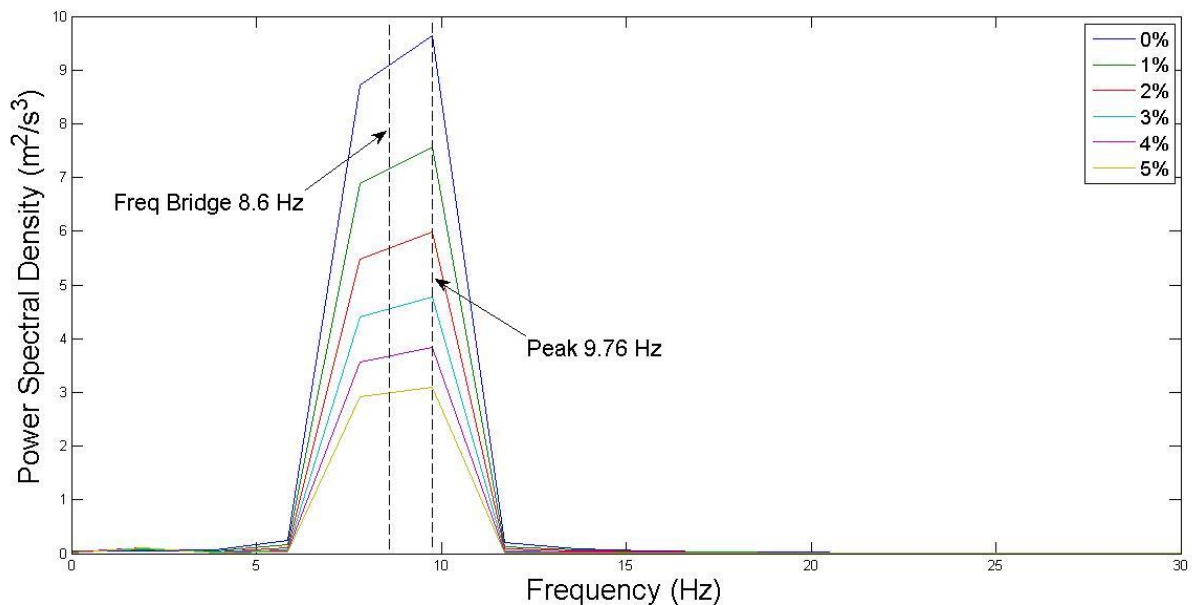


Fig. 3 Acceleration spectra for quarter car travelling at 20m/s on a 10m smooth bridge

For all the simulations with the smooth road profile, a peak is obtained near to the first natural frequency of the bridge – within the range of spectral resolution. Spectra obtained for

the other combinations of vehicle velocity and bridge span give similar results and are therefore omitted here. The resolution of the spectra is better for longer bridges as more data is obtained due to longer simulations. Generally, greater velocities and shorter spans reduce the time for the vehicle to record data as it crosses the bridge. Hence the frequency peaks are most accurate for the 35 m bridge and least accurate for the 10 m bridge. For all the frequency peaks in the spectra, a decrease in peak magnitude occurs for an increase in structural damping of the bridge. However, a smooth profile does not account for the true effect of a road profile on the vibration of the vehicle.

A rough road profile is also included in the simulations. The road irregularities of this profile are randomly generated according to ISO (International Organisation for Standardisation)(8608:, 1995) for a ‘very good’ profile or road class ‘A’. As for the smooth profile simulations, the structural damping is varied along with the vehicle speed and bridge span. The scanning frequency used in all simulations is 1000 Hz.

The power spectrum of the processed accelerations of the vehicle model travelling at 20 m/s over a 15 m is illustrated in Fig. 4. The spectra of accelerations obtained at all the simulated damping levels are also illustrated. There is no peak here corresponding to bridge frequency and there is no clear distinction between different levels of damping (all six graphs are on top of one another). The vibration of the vehicle dominates all of the spectra due to the rough road profile. This is because the ratio of height of road irregularities to bridge deflections is too large for the bridge to have a significant influence on the vehicle. Results for all combinations of vehicle velocity and bridge span yield similar results.

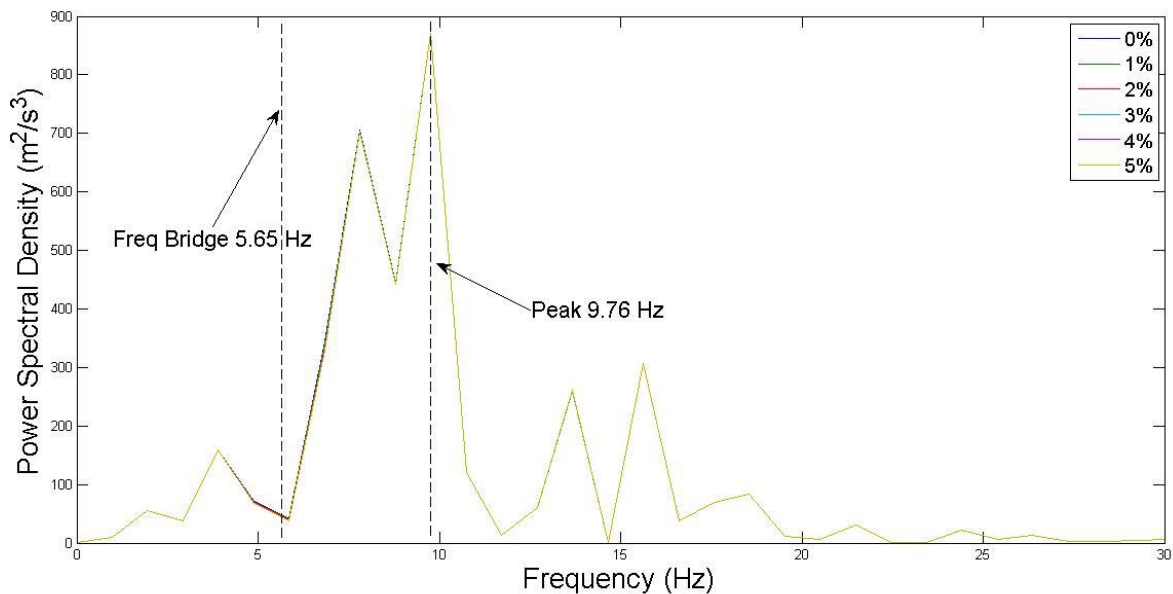


Fig. 4 Acceleration spectra for quarter car travelling at 20m/s on a 10m rough bridge

## RESULTS OF THE HALF CAR SIMULATIONS

The simulations in this section are performed using the half-car vehicle model with a rough road surface profile. The scanning frequency used in all simulations is 1000 Hz. The properties that are varied in these simulations are again the bridge span, bridge structural



damping and vehicle velocity. Having already obtained the bridge frequencies (Table 3), one can locate and differentiate between them in the acceleration spectra and compare recorded frequencies to the tabulated values.

When looking at the individual axles of the half-car (front and back) similar results can be seen as for the quarter-car model running over a rough profile. The vehicle axle frequency peaks dominate the spectra and it is extremely difficult to distinguish between the different levels of damping. However, when the axle accelerations are subtracted from one another, allowing for the time shift, clear peaks become visible corresponding to the natural frequency of the bridge. The power spectrum of the processed accelerations of the half-car vehicle model travelling at 20 m/s over a 35 m bridge can be seen in Fig. 5, where the front and rear axle accelerations have been subtracted from one another. A clear peak is visible at 2.93 Hz which corresponds to the first natural frequency of the bridge (Fig. 5). The inaccuracy is due to the resolution of the spectra ( $\pm 0.48$ ), which can be improved by driving the vehicle at a slower speed. Also, it is clearly visible that the magnitude of the peak decreases for higher levels of damping. In effect, the subtraction of accelerations removes the influence of the road profile. Furthermore, a second peak can be seen in the spectra, which corresponds to the second natural frequency of the bridge (12 Hz).

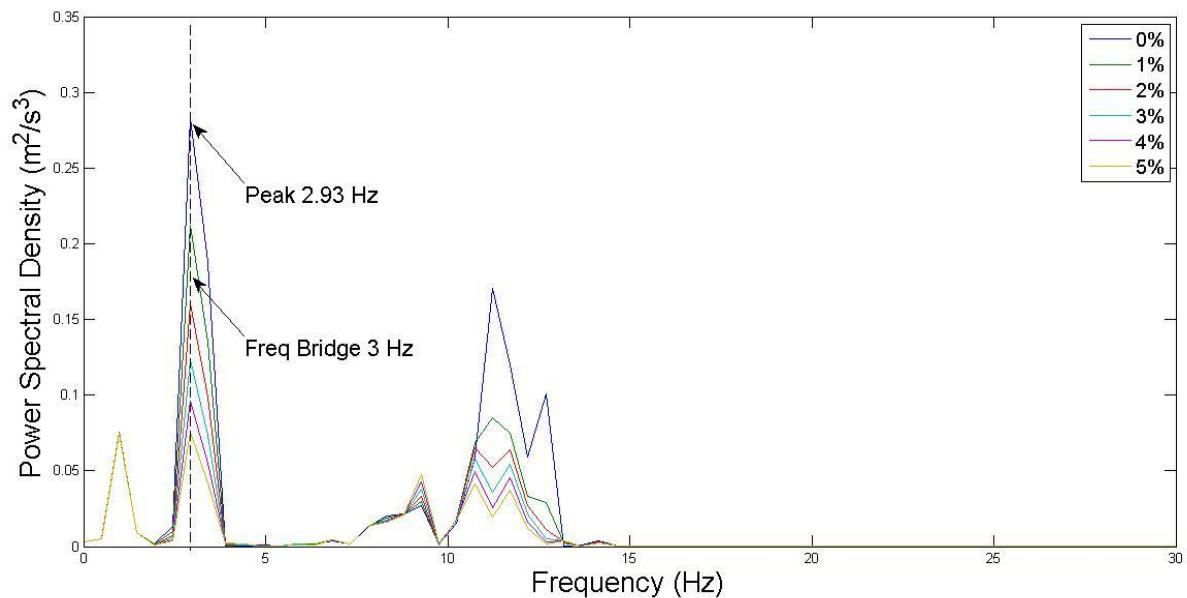


Fig. 5 Acceleration spectra for half car traveling at 20m/s on a 10m rough bridge

## SENSITIVITY TO VEHICLE VELOCITY

Fig. 6 shows the sensitivity of the peak power spectral density (PSD) to a 1% change in damping between 0% and 5% for the 10 m bridge. Values for all velocities are included here. For example, at 20 m/s the percentage decrease in peak PSD between 0% and 1% damping levels is represented by the first (dark blue) bar and has a value of 29%. At the same velocity, the percentage decrease in peak PSD between 2% and 3% damping levels is represented by the middle (green) bar and has a value of 20%. The percentage decrease in peak PSD is 12% between damping levels of 4% to 5% and is represented by the last (light blue) bar. From

this, a clear trend can be seen for a velocity of 20 m/s; the PSD is more sensitive to changes in damping at lower levels. This trend also exists for other velocities investigated. Also, there is a trend of decreasing sensitivity to damping for increasing velocities.

## SENSITIVITY TO BRIDGE SPAN

Fig. 6 shows the sensitivity of the peak power spectral density (PSD) to 1% changes in damping between 0% and 5% for a velocity of 20 m/s. Values for all bridge spans are included here. For example, for the 25 m bridge, the percentage decrease in peak PSD between 0% and 1% damping levels is represented by the first (dark blue) bar and has a value of 32%. For the same bridge span, the percentage decrease in peak PSD between 2% and 3% damping levels is represented by the central (green) bar and has a value of 23%. The percentage decrease in peak PSD is 18% between damping levels of 4% to 5% and is represented by the last (light blue) bar. A clear trend can be seen for a bridge span of 25 m; the PSD is more sensitive to changes in damping at lower levels. This trend also exists for other velocities investigated. There seems to be no clear trend in peaks for increasing or decreasing bridge span.

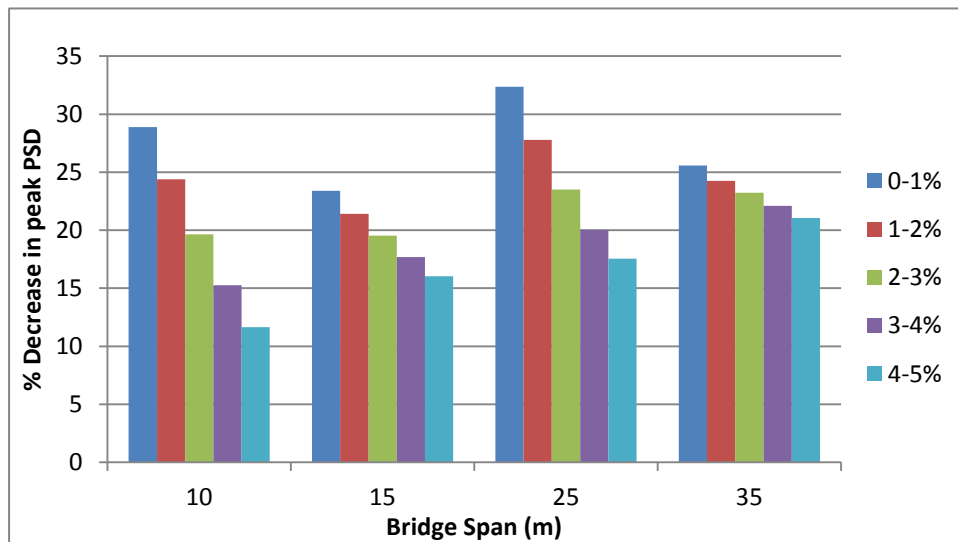


Fig. 6 Peak PSD-damping trends at bridge frequency peak for velocity of 20 m/s

## CONCLUSION

This study has investigated the feasibility of using an instrumented vehicle to monitor the damping in a bridge. The results show that it is possible to detect bridge frequency and changes in damping for a quarter-car on a smooth road profile, but it is more difficult to do for the same vehicle on a rough road profile. The results also show that the half-car model has the ability to detect bridge frequency and changes in damping while driving over a rough profile, when the axle acceleration of the front and rear axles are subtracted from one another. This was the case for a variety of vehicle velocities and bridge spans. It should be noted that no allowance has been made in this study for errors in measurement or a lack of symmetry in vehicle properties.

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